SUBSYSTEM FOR PREDICTING FLEXIBLE PAVEMENT PERFORMANCE

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Advanced systems of pavement design and management require methods of predicting pavement performance. In particular, long-range highway costs and user benefits can be established only if the future serviceability of pavements is known. Such cost-benefit calculations are important to highway management for programming construction and resurfacing priorities, for finding optimal solutions in design, and for setting administrative policies. Thus, mathematical models are needed by which pavement performance can be predicted as a function of age. This paper suggests that pavement performance models can be developed for any geographic and environmental conditions. The approach has been formulated so that local experience gained from successful highway pavements can be used. In principle, pavement performance is affected by two major distress mechanisms: traffic loads and environmental conditions. Pavement distress due to traffic was modeled by using AASHO Road Test performance data; successful thickness designs in Ontario were analyzed by using elastic layer theory. This analysis and AASHO Road Test data led to the development of a model primarily reflecting traffic deterioration. Distress due to environment was modeled by using Brampton Test Road data. The resulting submodels for the two distress types were combined to give a final model that fairly accurately predicted the performance of Brampton Test Road sections.

•A PRIMARY objective of pavement management is to provide pavement surfaces with acceptable riding quality at the lowest possible cost. It is widely accepted that, under the same traffic loading, a weak pavement structure will deteriorate faster and become rough more quickly than a strong pavement structure. The strong structure will require thicker construction with higher quality materials and initially will be more costly than the weak structure. Thus, the benefits of the longer life attained by stronger pavement must be weighed against the higher construction costs incurred. The stronger pavement will provide a smoother ride for a longer time than the weaker pavement and thus will favorably affect user vehicle operation and time costs. Thus when considering alternative pavement designs, management must be able to predict the probable riding quality of a pavement structure as a function of its age.

The ability to predict riding quality at the design stage has become possible only recently, stemming from the results of the AASHO Road Test (1). An improved mechanistic and more generally applicable method has now been developed and is described in this paper. The term performance, as it is used here, is the history of the riding quality as it varies over a period of time. Since economic comparison of alternatives is considered within a time frame, the performance of overlays required to restore structural integrity and renew riding quality is also described.

Based on investigations by Phang (2, 3) and Lister (4), the performance of a flexible pavement structure can be predicted from its early deflection response to standard load tests. Jung and Phang (5) established a relationship between performance and

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traffic by applying principles of elastic layer analysis to the AASHO Road Test results. In this paper, the performance prediction model has been extended to account for the losses in riding quality induced by the geographic and climatic environment, similar to those conceived by Scrivner et al. (6, 7). Further, a method is proposed to account for the deterioration of pavement layers at the end of the performance life when the pavement needs rehabilitation by an overlay.

The subsystem for performance prediction uses principles of linear elasticity to calculate the subgrade surface deflection of a flexible pavement structure when it is acted on by a standard wheel load. The simple method of calculation as suggested by Odemark (8) produces results that correlate closely with subgrade surface deflection (5) calculated by more rigorous but complex analytical procedures using programs such as CHEVRON and BISTRO. This simple method of calculation makes it practical to investigate a large number of alternative strategies.

The sequence of steps in the subsystem is shown in Figure 1. In step 1, an equivalent gravel thickness is calculated. In step 2, the calculated (Odemark) subgrade surface deflection for this structure is determined for the standard load and for the specific subgrade. Step 3 shows the traffic load in terms of repetitions N_t of the standard axle at any time Y_t in years. In step 4a, traffic N_t is used to determine the riding comfort index (RCI) loss p_{Tt} due to traffic for the subgrade surface deflection calculated in step 2. In step 4b, the year Y_t from step 3 is used to estimate the RCI loss p_{Et} due to environment for the subgrade surface deflection calculated in step 2. In step 4c, the two RCI loss components p_{Tt} and p_{Et} are summed and plotted against time Y_t. The life Y of the pavement is obtained from this plot where the sum intersects the desired total loss in RCI. A similar step procedure is used to determine the life of the overlay.

In the following sections of this paper, details of each step are given, including explanations of quantification and calibration according to Ontario experience.

FLEXIBLE PAVEMENT PERFORMANCE INDICATOR

Pavement deflection has often been accepted by many design agencies as a fair indicator of the structural strength of a pavement. Empirical formulas have been developed (3, 9, 10) to predict this value that can normally be measured in the field by using a Benkelman beam. Generally, limiting values of this deflection have been used as criteria for a successful pavement.

Recently, much effort has gone into applying elastic layer theory to predict deflections and stresses and strains and to design pavements based on these predicted responses. However, for attainment of a rational pavement design procedure, it is necessary to identify which of these responses will adequately indicate the future performance of a pavement and to know the location in the pavement structure where this response will be critical.

In Ontario, a comprehensive elastic layer analysis of existing pavement design experience (5) has revealed that elastic deformation at the top of the subgrade is the best indicator of pavement performance. The underlying hypothesis is that repeated subgrade deflections result in permanent deformation of the subgrade surface, which eventually leads to permanent deformation of the pavement surface and a decrease in riding quality.

Subgrade deflections may be calculated by using the procedure developed by Odemark (8), whereby an elastic layer system can be transformed into an equivalent granular thickness system by means of equivalency factors. The computed equivalent thickness H_e is then used to calculate the deflection w at the top of the subgrade as shown in Figure 2. Layer equivalency factors for determining equivalent granular thickness as shown in Figure 2 have been obtained from experience gained at the Brampton Test Road.

According to the Odemark method, certain properties of layer materials, named elastic moduli, are required for calculating subgrade deflections (8). A specific set of these properties was arrived at through a comprehensive analysis of experiences on various classes of roads in Ontario (5). These are shown in Figure 2. Layer

Figure 1. Steps in design subsystem.

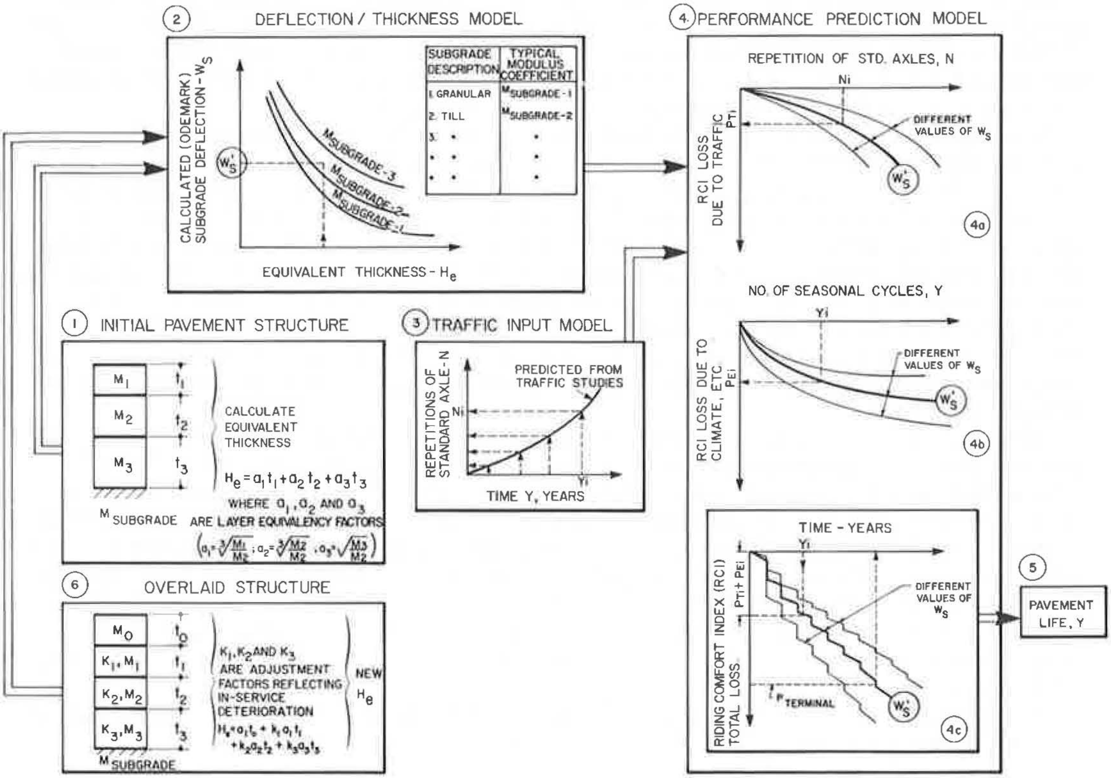
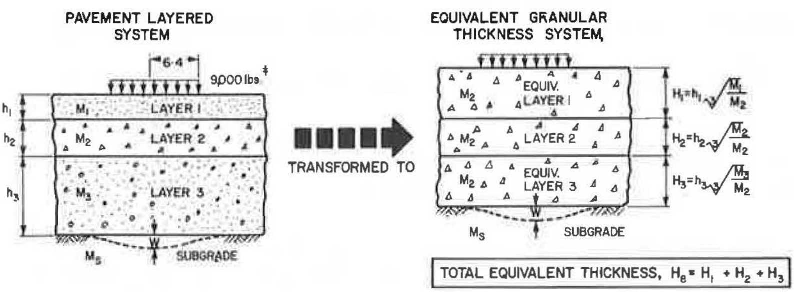


Figure 2. Transformation of elastic layer system into equivalent granular thickness system and calculation of subgrade deflection.



TOTAL EQUIVALENT THICKNESS, $H_e = H_1 + H_2 + H_3$

SUBGRADE DEFLECTION, $w = \dfrac{9{,}000}{2M_s\, Z\sqrt{1+\left(\frac{6.4}{Z}\right)^2}}$ [INCH]*

(UNDER A 9000lb. WHEEL LOAD)

WHERE

$Z = 0.9\, H_e \sqrt[3]{\dfrac{M_2}{M_s}}$ [INCH]*

M_s = SUBGRADE LAYER COEFFICIENT

M_2 = GRANULAR LAYER COEFFICIENT

* – 1 IN. = 25.4 mm

‡ – 1 LB. = 4.448222N

LAYER EQUIVALENCY FACTORS (ONTARIO)
SURFACE : BASE : SUBBASE = $\sqrt[3]{\frac{M_1}{M_2}} : \sqrt[3]{\frac{M_2}{M_2}} : \sqrt[3]{\frac{M_3}{M_2}} = 2:1:\frac{2}{3}$

THE 'SET' OF LAYER COEFFICIENTS ESTABLISHED FOR ONTARIO
M_1 = 400,000 ; M_2 = 50,000 ; M_3 = 15,000 ; AND M_s = 2,700 TO 11,000

equivalency factors as determined at the Brampton Test Road helped to establish the ratios among the values of these layer moduli (Figure 2). Since a fixed value of layer modulus is assigned to each layer, these values will henceforth be called layer coefficients to avoid confusion with the traditional definition of a modulus. A close scrutiny of the deflection equation using various other sets of layer coefficients has revealed that absolute values of layer coefficients are immaterial as long as their ratios remain constant (since ratios determine layer equivalency factors).

The hypothesis that subgrade deflection is a good indicator of pavement performance and that this deflection can be determined adequately by the set of layer coefficients as given in Figure 2 is substantiated in Figures 3 and 4. Figure 3 shows an excellent correlation between Odemark subgrade deflection calculated for AASHO Road Test sections (5) (using the set of layer coefficients in Figure 2) and the number of 18-kip (80-kN) equivalent applications successfully carried on these test sections to a terminal performance level of 2.5. Figure 4 shows a correlation that was observed at the Brampton Test Road between calculated Odemark subgrade deflections (using the same set of layer coefficients) and the measured Benkelman beam surface deflections. This response, from Canadian experience, is considered to be a good indicator of pavement performance (10).

Figure 5 is based on the established set of layer coefficients and shows a graphical representation of the Odemark subgrade deflection due to a standard 9-kip (40-kN) wheel load for various subgrade layer coefficient values (Table 1). Inherent in the curves of the figure are the following layer equivalency factors of surface:base:subbase = 2:1:$^2/_3$. The deflection w is obtained from this figure after a value is selected for the subgrade layer coefficient (Table 1) and after the value of the total equivalent granular thickness H_e for a pavement structure is calculated by using the above layer equivalency factors. On the right side of Figure 5 is a scale from which corresponding peak Benkelman beam surface deflections can be predicted as determined from the correlation established in Figure 4.

The method in the following sections shows how the Odemark subgrade deflection w is used to predict performance of a pavement structure.

PERFORMANCE PREDICTION MODEL

The performance prediction model in this paper includes the following:

1. A traffic input model that establishes the amount of traffic during the analysis time period (Figure 1, step 3),
2. A model for predicting the performance loss p_T due to traffic loads (Figure 1, step 4a),
3. A model for predicting the performance loss p_E due to geographic or environmental influences (Figure 1, step 4b), and
4. A method for combining p_T and p_E (Figure 1, step 4c).

Immediately after construction, the riding comfort index p_o of a pavement is relatively high and depends largely on the conditions during construction. From this value, the losses p_T and p_E must be subtracted to obtain the riding comfort index p at any particular time. Thus,

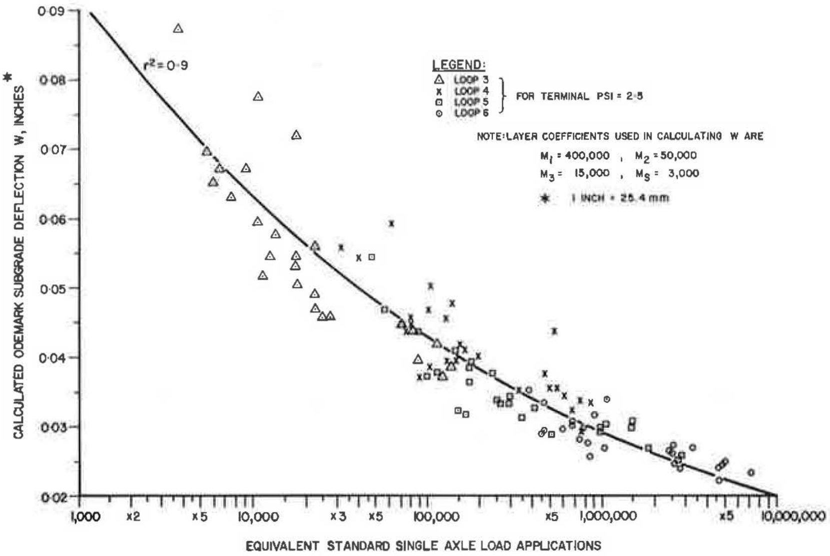
$$p = p_o - p_T - p_E \tag{1}$$

where

p_T = traffic loss, a function of the number of equivalent standard axle load applications N;

p_E = environmental loss, a function of the number of years or seasons passed Y; and

Figure 3. Calculated Odemark subgrade deflection versus load repetition at AASHO Road Test.



N = function of Y as established by the traffic input model.

Traffic Input Model

The traffic input model (Figure 6) is basically a functional relationship between traffic load N and time Y. The relationship can be established from traffic data (11).

The following method is being used in Ontario. Each project to be designed has a traffic estimate, usually an annual average daily traffic (AADT) in the design (first) year and a predicted AADT at the end of 20 years. First, the total equivalent standard [18-kip (80-kN)] single-axle load applications N_f expected during the analysis period A_p are determined by using an average of initial and final estimates of the number of 18-kip (80-kN) axles per day in the design lane as described by equation 3 in the following equations:

$$N = \frac{N_f}{A_p}\left[\frac{2AADT_i}{(AADT_i + AADT_f)}Y + \frac{AADT_f - AADT_i}{A_p(AADT_i + AADT_f)}Y^2\right] \tag{2}$$

$$N_f = \frac{A_p}{2}\left[\left(\frac{AADT_i}{2} \times DAYS \times T_i \times LDF_i \times TF_i\right) + \left(\frac{AADT_f}{2} \times DAYS \times LDF_f \times TF_f\right)\right] \tag{3}$$

where

T = percentage of trucks,
AADT/2 = one-directional AADT,
LDF = lane distribution factor (1.0 for two-lane roads, 0.8 for most four-lane roads),

Figure 4. Calculated Odemark subgrade deflection versus largest Benkelman beam deflection in spring, Brampton Test Road.

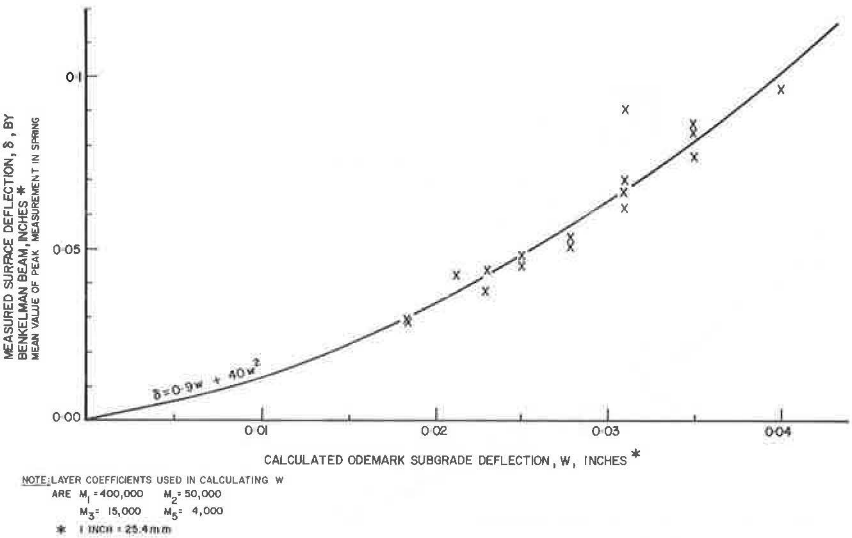


Figure 5. Odemark subgrade surface deflections for different equivalent granular thicknesses and subgrade layer coefficients.

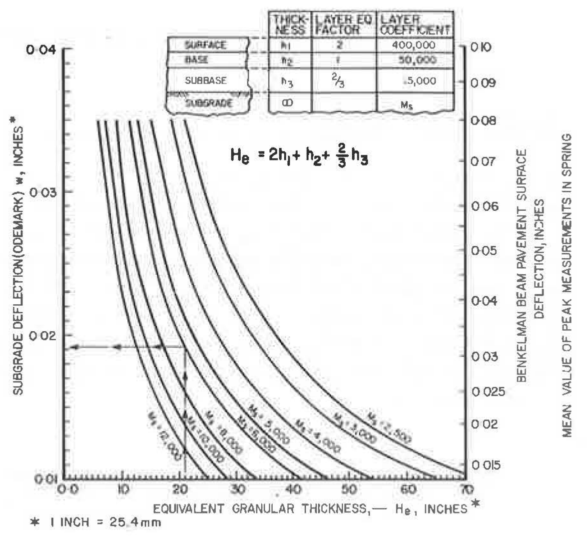
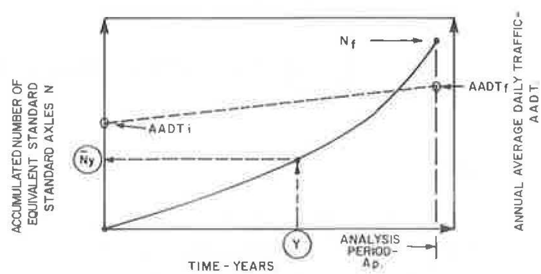


Table 1. Typical subgrade layer coefficients in Ontario.

Material	Condition		
	Good	Fair	Poor
Subgrade[a]			
Suitable as granular borrow	11,500	10,500	9,000
Silt <40, very fine sand and silt <45	7,000	6,000	5,500
Silt 40 to 50, very fine sand and silt 45 to 60	6,000	5,000	4,500
Silt >50, very fine sand and silt >60	4,500	4,000	3,500
Clay			
Lacustrine	5,500	5,000	4,000
Varved and Leda	4,500	3,500	2,500

[a]Granular type of sandy silt and clay loam till.

Figure 6. Traffic input model.



TF = truck factor, the equivalent standard axles per truck [18 kip (80 kN)],
DAYS = number of days per year for truck traffic (generally = 300), and
i and f = initial and final respectively.

Subsequently, at any number of years Y, the number of corresponding standard load applications N are determined by using Equation 2.

Traffic-Related Deterioration

The calculated Odemark subgrade deflection w is used to derive a performance prediction equation based on the main factorial test data of the AASHO Road Test (1). The equation [equation 11, (5)] has been rescaled from a 5-point present serviceability index (PSI) to a 10-point RCI. The modified performance prediction equation is given below and is shown in Figure 7.

$$p_T = 2.4455\,\psi + 8.805\,\psi^3 \tag{4}$$

where

ψ = 1,000 w^6 N in inches,
p_T = loss in RCI due to traffic,
w = calculated Odemark subgrade deflection in inches, and
N = number of standard [18-kip (80-kN)] single-axle load applications.

It should be noted that AASHO Road Test data did not include sections giving Odemark subgrade deflections of less than 0.025 in. (0.635 mm)(i.e., very thick pavements); however, the model can be extrapolated for designs in this range.

Environment-Related Deterioration

Since the AASHO Road Test was carried out with accelerated loading and relatively underdesigned pavement thicknesses, results are not readily applicable to real-life pavements even if such pavements were influenced by similar geographical and environmental conditions. Because of the exposure to climatic cycles, the real-life pavements suffer deterioration over a period of time, even under low traffic volumes, because of differential heaving and settling from frost, freeze-thaw cycles, swelling subgrades, or other such influences. Since the AASHO Road Test sections survived only one or two winter seasons, the test results do not adequately indicate the effect of these influences. This became apparent in Ontario when the traffic-related performance model derived from AASHO Road Test data was applied to the Brampton Test Road over a period of 8 years of its performance. The traffic performance model could not reproduce the Brampton results even when a proportionality factor was applied, and the characteristics of the difference suggested a different kind of functional relationship.

A flexible pavement attains its highest level of serviceability of riding comfort immediately after construction. During its lifetime, the pavement is exposed to forces from axle weights, temperature changes that cause restrained expansion or contraction, and changes of subgrade conditions that cause swelling, shrinkage, freezing-thawing, and heaving-settling. These forces or influences act in various combinations and proportions on a pavement, causing loss in the RCI.

The combination of axle weights with soft subgrade conditions in the spring and the upward migration of water in the subsoils due to freezing (resulting in heave or upward expansion and subsequent reconsolidation) are noteworthy distress mechanisms. The influence of spring softening observed in the AASHO Road Test was the reason why the number of axle repetitions were weighted. The effect of water migration and freezing is a function of the increasing number of years through which a pavement survives. In addition, deterioration due to temperature changes (such as transverse cracking) is a function of the number of winter-spring seasons. Because of this time-dependent part of pavement deterioration, a loss model, in addition to a traffic loss model, should be developed as a function of number of years or winter-spring cycles.

The function that best represents this additional loss term is conceived to have the specific characteristic of decreasing exponentially with each year or season passed. A similar function has been suggested by Kher, Hudson, and McCullough (12) in the development of pavement performance equations for swelling clay subgrades in Texas, a condition that produces similar volume changes as those produced by low-temperature frost effects. The following equation

$$p_E = \left(p_\circ - \frac{7.5}{1 + Bw}\right)\left(1 - e^{-\alpha Y}\right) \quad (5)$$

where

w = Odemark subgrade deflection in inches,
B = 60, and
α = 0.06.

shows that the rate of decrease in p due to environmental forces is at a maximum in the initial years and reduces with time as p reaches a hypothetical ultimate value of p_∞ at infinite time (Figure 8). In other words, the more a pavement deteriorates from its initial smoothness toward its ultimate roughness value of p_∞, the less the annual rate of deterioration will be. For regions where a different type of environmental deterioration is observed, the loss function may assume a different form such as, for example, a linear relationship that should be used where the same average amount of environmental deterioration is observed in each year of pavement life.

It is also conceived that the level of p_∞ is larger for stronger pavements, i.e., stronger (thicker) pavements will be less affected by environmental deterioration than weaker pavements. Since the strength of a pavement has been established in the previous section by the subgrade surface deflection criterion, the same response value can be used to establish the resistance of pavements to environmental forces. In other words, p_∞ can be made a function of the Odemark subgrade deflection.

To establish the constants of the exponential deterioration function requires use of experimental data or other experience on real pavements in a particular region. In Ontario, the Brampton Test Road data were used for this purpose. The methodology adopted to establish this function is described below; however, it is obvious that each region must rely on local experience to obtain a relevant function with different constants.

For each section of the Brampton Test Road, the best fit curves and the curve predicted by the traffic performance model were drawn through the observed data. The two curves did not coincide because of the differences contributed by the environmental factors. When the differences were examined for all the sections of the Brampton Test Road, three inferences were drawn.

1. Environmental deterioration is an exponential curve of RCI versus the number of years.
2. Exponential deterioration rate of α = 0.06 applies to all sections of the Brampton Test Road.
3. Asymptotic value of p_∞ varies for each section and correlates highly with the strength of the section represented by the Odemark subgrade deflection w. The correlation is

$$p_{\infty} = (p_o/1 + Bw) \tag{6}$$

where p_o is initial RCI of pavement, and B = 100.

Figure 9 shows the basic performance data on some of the Brampton Test Road sections, ranging from very weak to very strong pavements. The curves, calculated by the equations of the final model that combine deterioration resulting from both traffic and environmental influences (using the above constants), are also shown. These curves demonstrate that the final model predicts the riding comfort measurements of the Brampton Test Road over the entire range of conventional pavements tested. Also shown are the curves resulting from only the traffic deterioration model.

Data on the values of RCI taken immediately after construction indicated that average p_o for the pavements can be assumed to be equal to 7.5. With p_o at a constant value, the model was tested on a variety of past pavement design experiences in the province. The analysis showed that, for B equal to 60, pavement lives matched the province-wide experience.

EXAMPLE OF MODEL APPLICATION

The following example is presented to demonstrate how the effects of traffic and environmental loss in RCI are combined and used to predict the age at which terminal performance is attained.

Assume that there is a pavement with 3½-in. (8.9-mm) asphalt surfacing, 6-in. (152-mm) granular base, and 12-in. (305-mm) granular subbase constructed on a soft clay subgrade and that the estimated number of standard 18-kip (80-kN) axle load repetitions at 12 years is 300,000. The first step in the procedure is to calculate the total granular equivalent thickness H_e of the pavement structure. This is $(3\frac{1}{2} \times 2) + (6 \times 1) + (12 \times \frac{2}{3}) = 21$ in. (533 mm) when the appropriate layer equivalency values are applied to each layer.

For the second step, the subgrade layer coefficient selected from Table 1 as being consistent with the description of the material is M_s = 2,700. The Odemark subgrade surface deflection w for this pavement under the standard 9-kip (40-kN) wheel load P [contact radius, a = 6.4 in. (163 mm)] is calculated by

$$w = \frac{9{,}000}{2M_sZ} \times \frac{1}{\sqrt{1 + \left(\frac{a}{z}\right)^2}} \tag{7}$$

where

$$Z = 0.9 \sqrt[3]{\frac{M_2}{M_s}} \times H_e \tag{8}$$

or the deflection w may be read directly from Figure 5.

For this example M_s = 2,700; M_2 = 50,000 (adopted for the subsystem), and H_e = 21 in. (533 mm). Z is calculated as 50 in. (1270 mm), and w is calculated as 0.03306 in. (0.84 mm).

For the third step, the number of standard axles N at any time Y must be known. For the example, N = 300,000 at Y = 12 years. The loss in riding comfort index, p_T due to traffic is determined from the following equation:

Figure 7. Performance prediction model, traffic-related contribution.

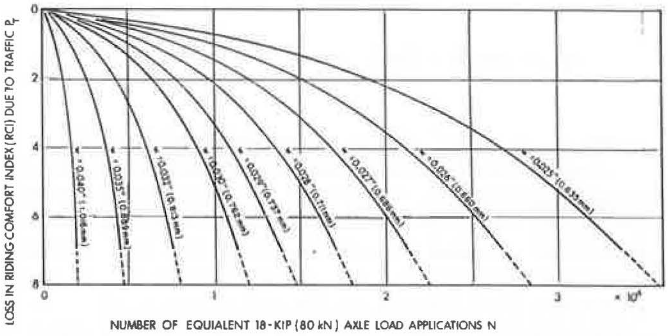


Figure 8. Performance prediction model, environment-related contribution.

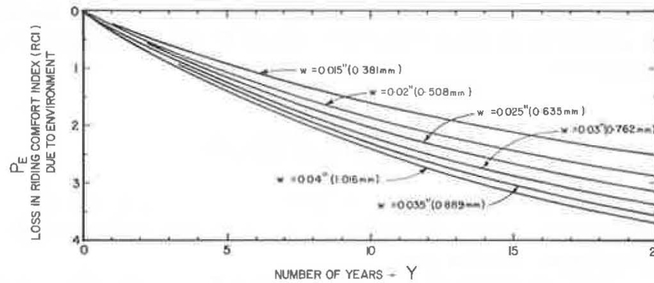
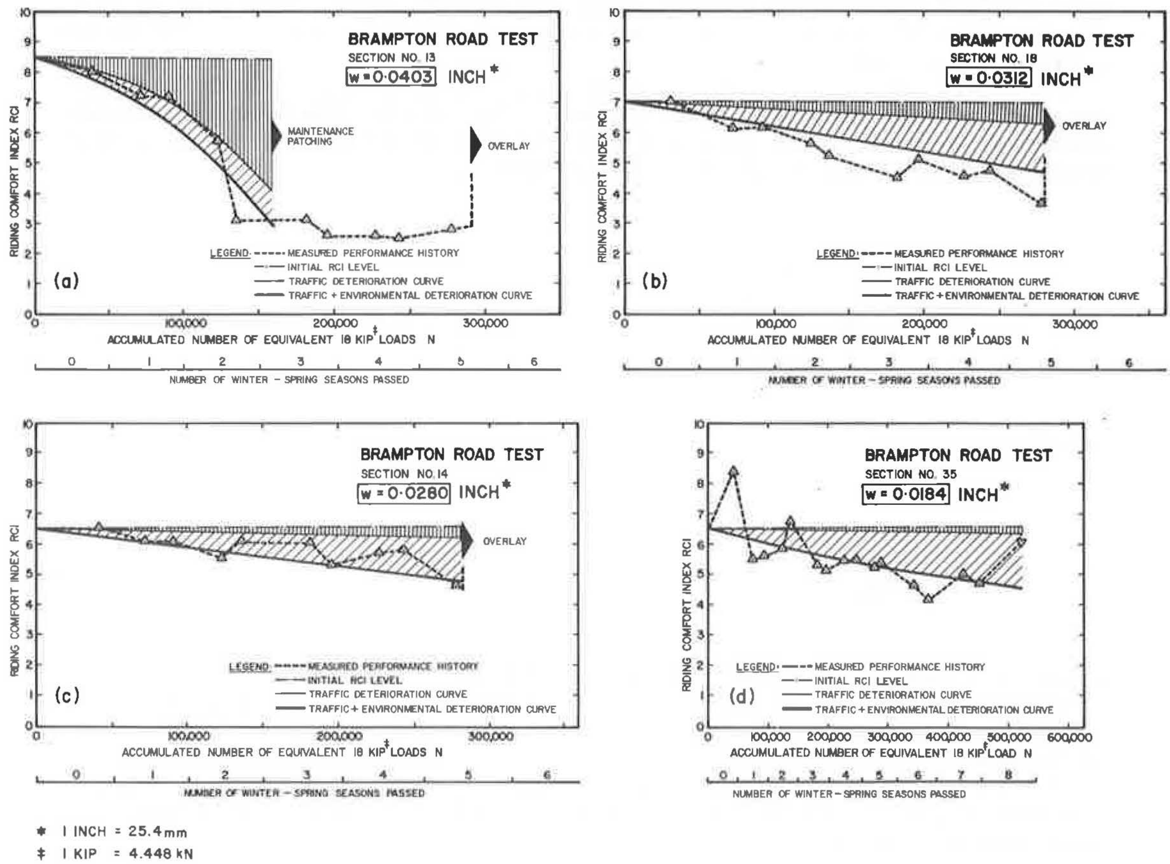


Figure 9. Brampton Test Road performance data versus predicted performance based on model developed.



* 1 INCH = 25.4 mm

‡ 1 KIP = 4.448 kN

$$p_T = 2.4455\psi + 8.805\,\psi^3 \tag{9}$$

where

ψ = 1,000 $W^6 \times N$ in inches,
ψ = 0.3917, and
p_T = 1.487.

The RCI loss may also be read from Figure 7.

For the fourth step, the loss in RCI due to environmental influences p_E is calculated by the following formula:

$$p_E = (p_o - p_\infty)(1 - e^{-\alpha Y}) \tag{10}$$

where

$$p_\infty = (A/1 + Bw) \tag{11}$$

For equation 10, as is the case with most new construction, the initial RCI value p_o is 7.5. The constants in these equations are $p_o = 7.5 = A$, $B = 60$, and $\alpha = 0.06$. Use of the values of B and α for southern Ontario appears to predict lives that fit experience. In addition, $p_\infty = 2.514$, and $p_E = 2.559$. This may also be read from Figure 8.

For the fifth step, the RCI losses p_T and p_E are subtracted from the initial riding comfort index p_o to determine the RCI at 12 years, or $p_{12} = p_o - p_T - p_E = 7.5 - 1.487 - 2.559 = 3.454$. This predicted RCI at 12 years forms one point on the performance curve for this pavement structure. Other points are similarly calculated for different times Y and their corresponding traffic loads N so that the performance curve may be defined. When this is done as is shown in Figure 10, the age of the pavement may be determined for any value of RCI that is considered to be the terminal value or the value at which rehabilitation becomes necessary (Table 2). For example, at a terminal RCI of 4.0, the corresponding predicted age taken from the performance curve is 10.8 years.

OVERLAYS

Repeated traffic loads and environmental influences result in gradual deterioration of pavement layers. Cracks develop in the asphalt layers and increase in number over the years; concurrently, the granular layers are weakened by various mechanisms such as aggregate degradation and contamination.

Thus, although the life of a newly constructed pavement is predicted based on its initial value of calculated subgrade deflection w, this deflection value does not remain constant but increases as the strength of the pavement layers decreases. This increase in deflection was observed at the Brampton Test Road (13) and demonstrated by Lister (4) in his observations on deflection of more than 300 experimental sections. For the design of an overlay, the reduction in the strength of the existing layers can be accounted for by reducing their layer equivalency factors. The reduction coefficients that are used at present in the subsystem are shown in Figure 11.

The premise underlying the use of layer equivalency reduction coefficient K_1 for asphalt layers is that repeated loading decreases stiffness (14); thus, the coefficient K_1 decreases as RCI decreases to values slightly below that of poor gravel. A straight line relationship was considered most suitable. Figure 11 similarly shows layer equivalency reduction coefficients K_2 and K_3 for granular base and subbase materials.

The coefficients are larger for stronger pavements because such pavements are designed to withstand heavier traffic loads or more seasonal cycles. The layer equivalency reduction coefficients, although based on limited data, not only serve as indicators of how the modeling of the overlay problem is approached, but also result in overlay lives that are acceptable to practicing design engineers in Ontario.

An overlay rehabilitates the pavement in two respects. First, it raises the riding quality to a more comfortable level (Figure 12) and, secondly, it reestablishes the strength of the pavement. As a general guideline, overlay thickness should compensate for some of the loss in strength of the old pavement so that the total granular equivalent thickness after the overlay is comparable with the thickness of the initial construction. The equivalent thickness after overlay is determined as follows:

$$H_e = a_1h_o + k_1a_1h_1 + k_2a_2h_2 + k_3a_3h_3 \tag{12}$$

where

h_o = overlay thickness;
a_1, a_2, a_3 = layer equivalency factors for asphalt surfacing, base, and subbase respectively;
h_1, h_2, h_3 = layer thicknesses of asphalt surfacing, base, and subbase of initial pavement respectively; and
k_1, k_2, k_3 = layer equivalency reduction factors (Figure 11) for asphalt surfacing, base, and subbase respectively.

For prediction of the life span of an overlaid pavement, the new equivalent thickness value replaces the initial equivalent thickness value, and the performance prediction procedure is repeated.

CONCLUSIONS

A model has been described that predicts the performance life of a pavement structure. Alternatively, if several pavement structures are proposed for a facility, the most suitable may be selected based on performance life and cost implications.

The model has been formulated so that local experience gained from successful highway pavements can be used. Basically, the performance of a pavement has been considered to be affected by two major distress mechanisms: traffic loads and environmental conditions. Traffic-related distress has been derived from the AASHO Road Test, which is primarily a traffic-oriented test. Environment-related distress has been derived from the Brampton Test Road, which gives the long-term effects of traffic and the environment. The two tests have been analyzed in relation to past pavement design experience, which has been given prime consideration in the curve-fitting process.

The model has undergone considerable testing with its application for designing pavement structures throughout Ontario. The lives predicted by the model are found to be in close agreement with the general pavement performance experience of the design engineers.

As an empirical formulation, the model contains significant lack-of-fit error due to the nature of pavement performance data, which inherently vary even among identical pavement sections. The lack of fit generally implies that more variables affect pavement performance than those considered in the study; however, the model does predict average life expectancy, and the lack of fit is even smaller than that found on the AASHO Road Test. In the range of extremely low deflections (i.e., extremely thick pavement structures), AASHO Road Test data did not exist. However, the use of the extrapolated curve outside of the AASHO range is currently considered appropriate.

The procedure in this paper can be used to develop performance models for other regions. Since pavement deterioration that is generated by traffic alone is less

Figure 10. Predicted performance curve.

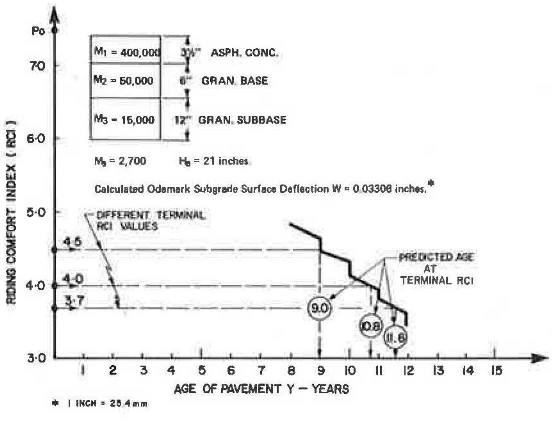


Table 2. Predicted ages at different terminal riding comfort index values.

Age	No. of Standard Axles	RCI Loss Traffic-Related	RCI Loss Environment-Related	RCI at 12 Years
8	2.0	0.795	1.900	4.805
9	2.25	0.942	2.080	4.478
10	2.50	1.104	2.250	4.146
11	2.75	1.285	2.409	3.806
12	3.0	1.487	2.559	3.454

Figure 11. Reduction coefficient for layer equivalency factor.

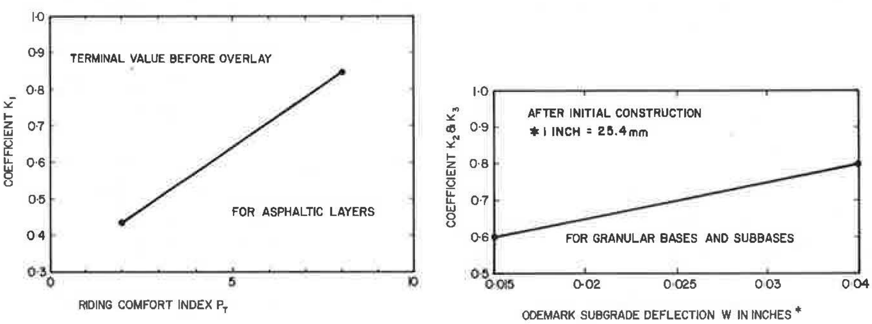
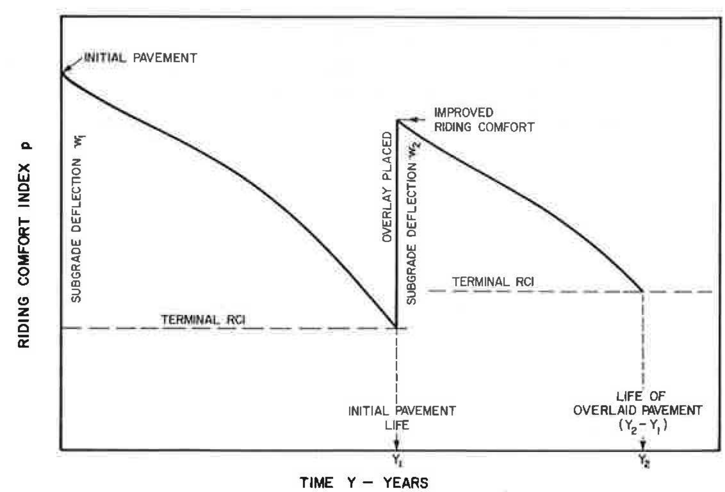


Figure 12. Overlay performance concept.



significant than that caused by environment, the traffic deterioration model in this paper can be adapted to other regions. Research effort can then be concentrated on updating the environmental model based on local experience.

The model is by no means final, and updating of the submodels, such as the environmental deterioration model, is continuing as more data become available.

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